Pioneer Recreation and Sports Centre

Detailed Engineering Evaluation
Stage Two Quantitative Assessment Report

75 Lyttelton Street, Somerfield, Christchurch

Christchurch City Council

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Executive Summary

Christchurch City Council appointed Opus International Consultants to carry out a detailed seismic assessment of the Pioneer Recreation and Sports Centre, located at 75 Lyttelton Street, Christchurch. The key outcome of this assessment was to ascertain the anticipated seismic performance of the structure and to compare this performance with current design standards. Opus were also asked to provide conceptual strengthening options to improve the building's seismic performance, with a target of meeting at least 67% of the new building standard (%NBS).

The Qualitative report carried out by Opus in September 2011 [1] identified four critical structural weaknesses (CSW) but concluded that there were mitigating factors and had not caused damage due to the February 2011 earthquake, these were . These are discussed in more detail in Section 6.1.

The detailed assessment identified a small pier at first floor level in the former squash courts to have a minimum 73%NBS capacity. Neglecting this detail the seismic capacity of the building has been calculated as approximately 84% NBS, including all critical structural weaknesses.

The stadium portal frames have a snow loading capacity of 89% NBS.

Minor repair works should be carried out on cracking to reinforced concrete masonry walls in the Stadium and opening of the slab joint in the foyer.

It is considered that the building is economic to repair.

1 Introduction

Opus International Consultants Limited has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Pioneer Recreation and Sports Centre, located at 75 Lyttelton Street, Somerfield, Christchurch.

A qualitative assessment of the building in September 2011 [1] identified that the seismic capacity of the building was around 50%NBS based on the reinforced concrete masonry walls of the original parts of the structure. The qualitative assessment also identified several Critical Structural Weaknesses including an apparent lack of bracing/diaphragm to the Pool Hall roof. A limited quantitative assessment was recommended in order to determine whether the building has a seismic rating of at least 67%NBS, which is the recommended performance standard for existing buildings [2].

Preliminary calculations carried out as part of the qualitative assessment also identified that the steel portal frames of the original Stadium are governed by snow loading rather than seismic forces, with an estimated snow capacity of 30%NBS.

The scope of the quantitative assessment is as follows:

1. An analysis of the seismic capacity of the reinforced concrete masonry walls within the original parts of the structure;
2. A more detailed assessment of the steel portal frames of the original Stadium under snow loading;
3. A more detailed consideration of the Pool Hall roof structure and load paths to transfer seismic forces to ground level;
4. To identify any improvement work necessary to bring the building up to the minimum 67%NBS.

2 Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee to carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.
2. The placard status and amount of damage.
3. The age and structural type of the building.
4. Consideration of any critical structural weaknesses.

We anticipate that any building with a capacity of less than 33% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building Policy.

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 - Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to the alteration.

This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council) is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or

2. In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or

3. There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or

4. There is a risk that that other property could collapse or otherwise cause injury or death; or

5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property.

A moderate earthquake is defined by the building regulations as one that would generate loads 33% of those used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake on 4th September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012.
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone.
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and
4. Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

1. The accessibility requirements of the Building Code.
2. The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

1. 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3)
2. Increased serviceability requirements

3 Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The loadings are in accordance with the current earthquake loading standard ASNZS1170.5 [3].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented in Figure 1 below.

Table 2.2 NZSEE Risk Classifications and Improvement Recommendations

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance: Legal Requirement	Improvement of Structural Performance: NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)	Unacceptable	Unacceptable

Figure 1: NZSEE Risk Classifications Extracted from Table 2.2 of the NZSEE 2006 AISPBE Guidelines

Table 1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Table 1: %NBS compared to relative risk of failure

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

4 Background Information

4.1 Building Description

4.1.1 General

The Pioneer Recreation and Sports Centre is a complex of single level and two storey structures located at 75 Lyttelton Street. The original building, consisting of the Stadium, Administration Block and Squash Courts was constructed in 1977. The Pool, Gymnasium and Administration Extension were added in 1999 and in 2008 the squash courts were infilled with a floor at level 1 to create Fitness Studios. The 1999 works also included a separate Creche building adjacent to the Stadium.

The original building was oriented on an east-west axis with the stadium to the west and the squash courts on the east. The 1999 addition located the gymnasium to the east of the squash courts and the pool immediately to the north. Lyttelton Street lies immediately to the east of the Centre. For the purposes of this report we refer to the direction parallel to Lyttelton Street as the north-south direction.

The Stadium is a single level steel portal frame structure with part height reinforced concrete masonry walls around the perimeter, approximately 40 metres wide (north-south) by 56 metres long (east-west) in plan dimension. The Administration Block is a two storey structure consisting of reinforced concrete masonry internal and perimeter walls with a precast concrete double tee and in-situ concrete floor over. The roof is lightweight steel clad supported on steel portal frames above the level 1 slab. The old Fitness Studios consist of reinforced concrete masonry walls with lightweight roof and the infill concrete floor slab at level 1 supported on steel beams spanning in the east west direction. The Stadium and Fitness Studios ground floors are suspended timber construction.

The foundations consist of reinforced concrete piles connected by reinforced concrete ground beams. All reinforced concrete masonry is fully grouted.

The Gymnasium and Administration extension structure consists of reinforced precast concrete wall panels supporting the level 1 precast concrete flooring units with insitu concrete topping. The roof is lightweight steel framed and clad. The foundations are reinforced concrete piles and ground beams and concrete slab on grade floors.

The Pool Hall consists of an insulated sandwich panel roof supported on structural steel framing spanning in the east-west direction. The east and west walls consist of lightweight timber framing with lightweight cladding at high level and precast concrete panels at low level. The north and south walls are full height precast concrete panels. The floor slab and pools are constructed from reinforced concrete slab on grade.

The south wall is supported on reinforced concrete piles and ground beams, while the remainder of the Pool Hall structure is supported on reinforced concrete pad or strip footings.

The Creche Building is a single storey reinforced concrete masonry, structural steel and timber framed structure. The lightweight roof is supported on structural steel portals in the north south direction. The south wall (adjacent to the Stadium) is full height reinforced concrete masonry, supporting the steel portal rafters. The remaining perimeter and internal walls are timber framed.

4.1.2 Gravity load resisting system

The Stadium has seven bays in the east-west direction. Large 762UB steel portal frames span the width of the building. Intermediate columns to the west portal and a simple steel frame to the east form the end walls. The lightweight iron roof and wall cladding is supported on cold formed steel purlins and girts attached to the outer flange of the portal frame. The purlins and girts also support the internal ceiling and wall linings.

The portals are supported on reinforced concrete piles tied together with a reinforced concrete ground beam around the perimeter of the building. These perimeter beams also support the 3.2m high reinforced concrete masonry wall panels.

The Administration Block and Squash courts have a lightweight steel roof supported on structural steel portals. The portal frames are supported on the level 1 slab. The level 1 slab consists of 350mm deep precast double tee units with 65mm insitu topping. The 350TT units are flange supported with a minimum of 75mm seating on the ground floor reinforced concrete masonry walls. These walls are fully grouted with vertical reinforcement cast into the insitu topping above.

The reinforced concrete masonry walls are 190mm thick and create the Administration and changing room spaces. Openings in the walls are spanned by 400mm deep reinforced concrete beams, 380mm wide where support for the slab over is provided. The walls are supported by reinforced concrete ground beams and piles. The drawings appear to show that the reinforced concrete slab on grade to the Administration area is a floating slab, not connected to the walls or ground beams.

The Gymnasium and Administration Extension is similar to the original Administration Block. The differences in this area are that the walls are precast concrete panels instead of reinforced concrete masonry, the level 1 slab is 350TT and 75mm precast flat slab units with cast insitu topping, and the ground floor slab is supported on the ground beams.

The Pool Hall lightweight insulated roof panels span between structural steel portals in the east-west direction. On the east side, the steel portal frames extend down to the ground to be supported on shallow pad foundations. On the west side, the portal columns are supported on top of reinforced concrete columns that in turn are supported on a shallow strip footing. The east and west walls have precast reinforced concrete panels up to approximately 2.2m, with lightweight cladding above. The north and south walls and north plant room walls are full height precast concrete panels.

The Creche Building 200UB portal frame rafters are supported on the southern masonry wall and steel columns to the north, with internal 75SHS steel posts at approx one third span. Shallow strip footing support both the masonry wall an portals.

4.1.3 Seismic load resisting system

The seismic resisting system for the Stadium structure is the steel portals in the north south direction and 100x8 equal angle steel cross bracing in the (longitudinal) east-west direction. There are two bays of bracing on each side and the bracing is continuous across the roof (steel flat with steel PFC chords) in these bays also. The bracing extends down to ground level and so the reinforced concrete masonry perimeter walls are self supporting only. The walls span between the reinforced concrete ground beam and the bond beam at the top to resist face loads.

Above Level 1 the Administration and Fitness Studios lateral load resisting system consists of the steel portals in the north south direction, and in the east west direction is connected via the concrete wall on grid 5 to the cross bracing in the roof of the adjacent Aerobics gym. The level 1 reinforced concrete slab will act as a diaphragm to distribute forces to the reinforced concrete masonry walls acting as in-plane shear walls.

The Gymnasium and Administration Extension has 60x8 equal angle steel cross bracing in the roof to distribute loads to the perimeter precast concrete wall panels. The wall panels are 150mm or 200mm thick and are tied into the concrete slabs at level 1 and ground. The extension is separated from the original building by a 50mm wide seismic gap.

The Pool Hall seismic resisting system in the east-west direction is a hybrid of three structural steel portal frames and the north and south face perimeter precast concrete walls acting as in-plane shear walls. No roof bracing exists to tie these seismic resisting elements together and no connection details for the insulated panels are given to show that they are acting as a diaphragm. These elements will therefore act in an independent manner supporting their own tributary seismic mass.

In the north-south direction, the east and west walls act as in plane shear walls. Loads from roof level are transferred to the top of the precast concrete panels by steel cross bracing. The west wall has only a short length of precast panel, to which the loads are transferred by a concrete tie beam along the full building length at the top of the wall. Overturning of this wall section is resisted by the continuous strip footing below.

The Creche building seismic resisting system consists of reinforced concrete masonry walls on the south side and plywood lined timber walls on the north side acting as inplane shear walls in the east west direction. Seismic loads in the north south direction are resisted by the steel frames as portals.

4.2 Survey

A structural assessment of the building was undertaken on 17 August 2011 by Opus Senior Structural Engineer David Dekker. This inspection included external and internal visual inspections of all structural elements above foundation level, and of areas of damage to structural and non-structural elements.

4.3 Original documentation

Copies of the following construction drawings were located on site and copied;

- Centennial Park Sports Complex for Christchurch City Council, structural drawings with various dates from December 1976 to March 1977.

- Alterations to Pioneer Leisure Centre, architectural drawings dated February 2007 for Squash Court floor infill.

Copies of the following construction drawings were provided by CCC:

– Pioneer Sports Stadium Pool, Stadium Additions, Creche, architectural (Barclay Architects), structural (Holmes Consulting Group) and services (Keen and Beca) drawings dated December 1999.

No copies of the design calculations have been obtained as part of the documentation set.

5 Structural Damage

The following damage has been noted:

5.1 Perimeter concrete masonry walls to the Stadium

a) Cracking to reinforced concrete masonry in one location along the north, adjacent to a column. The crack had been most covered by plywood and was not able to be thoroughly inspected.

b) Opening of the joint between the reinforced concrete masonry walls in the north west corner.

5.2 Ground slabs

a) Minor horizontal separation of the Foyer ground level slabs at the junction of the existing building and the Administration extension.

6 Detailed seismic assessment

6.1 Critical structural weaknesses

As outlined in the Critical Structural Weakness and Collapse Hazards draft briefing document, issued by the Structural Engineering Society (SESOC) on 7 May 2011, the term ‘Critical Structural Weakness’ (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of the building. We have identified the following CSW’s for this building.

a) Stiffness irregularity – there is a significant difference between the lateral stiffness in the north south direction of the stadium portals and the Administration reinforced concrete masonry walls. This is mitigated somewhat by the height difference between the portals and the level 1 slab and also the use of steel portals above level 1 in the Administration area. However, increased levels of damage expected along the interface between these buildings has not been noted.

b) Stiffness irregularity – the walls in the north south direction of the Pool Hall are significantly different in length. However, as noted in Section 4.1.3 no roof bracing exists and the roof does not appear to act as a diaphragm. These elements will therefore act in an independent manner supporting their own tributary seismic mass.

c) Change in foundations – the south wall of the Pool Hall and the remainder of the Centre is supported on reinforced concrete ground beams and pile foundations while the remainder of the Pool Hall is supported on shallow pad or strip footings. The differing foundation conditions may give rise to different behaviour of this part of the centre with respect to settlement.

d) Built in stairs – the public stairs in the Administration area between ground floor and level 1 are tied in to the structure at both levels. However due to the rigid shear wall structure of the building the relative deflection between levels and potential for subsequent damage to the stairs is small.

6.2 Quantitative assessment methodology

6.2.1 Seismic forces

The following criteria from the earthquake loadings NZS 1170.5 [3] were used to determine the site loading spectrum:

Parameter	Value	Comments
$C_h(T)$	3.0	Class D soil, $T_1 < 0.5$secs
Z	0.3	Increased seismic hazard factor for Christchurch
R	1.3	Importance level 3 (based on the building having a space where more than 300 people can congregate)
N(T,D)	1.0	No near fault factor included

The seismic forces in the various components were calculated in accordance with the following ductility criteria:

Component	Ductility Factor μ
Reinforced concrete masonry walls, including bond beams subject to out of plane forces	2.0
Precast concrete walls resisting in plane shear forces	1.25

6.2.2 Material properties

The following material properties were used in the analyses:

Material	Nominal Strength
Structural steel	f_y = 250MPa(1)
Reinforcing steel	f_y = 380/275MPa(2) f_y = 460MPa(3)
Concrete	f'_c = 40MPa(3)
Masonry	f'_c = 10MPa(2)

(1) Stadium portal frame
(2) In stadium reinforced concrete masonry walls for high yield and mild steel respectively
(3) 1999 precast concrete walls

6.3 Limitations and assumptions in results

Our analysis and assessment is based on an assessment of the building in its undamaged state. Therefore the current capacity of the building will be lower than that stated.

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

1. Simplifications made in the analysis, including boundary conditions such as foundation fixity.
2. Assessments of material strengths based on drawings only which would assume good construction practices.
3. The normal variation in material properties which change from batch to batch.
4. Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

6.4 Quantitative assessment results

6.4.1 Seismic capacity

The equivalent static load method was used to analyse the seismic forces. The results of the analysis are reported in the following table as %NBS, where for the components:

%NBS = the reliable strength ÷ new building standard force

Structural Element/System	Failure mode or description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Stadium portal frames	Flexure	No	95% NBS
Stadium vertical cross bracing	Axial tension	No	>100% NBS
Stadium masonry walls in plane	Shear	No	>100% NBS
Stadium masonry walls out of plane including bond beam	Flexure	No	>100%NBS
Administration masonry walls out of plane	Flexure	No	>100%NBS
Administration/fitness studio North-South portal frames	Flexure	No	>100% NBS
Administration/fitness studio masonry walls in plane	Shear	No	>100% NBS
Former squash court masonry walls out of plane	Flexure	No	>100%NBS
Former squash court masonry wall pier	Flexure	No	73% NBS
Gymnasium/Administration extension precast concrete walls in plane	Shear	No	>100% NBS
Precast concrete wall on grid 7 in plane	Shear	No	>100%NBS
Pool Hall East-West frames on grids D, E & F	Flexure	No	>84%NBS(1)
Pool Hall East and West walls	Shear	No	>100% NBS

(1) Critical section 410UB54 columns on grid 1

The calculations for the East-West frames also identified a theoretical lateral eaves deflection under ULS seismic loading of around 200mm.

6.4.2 Snow load check on Stadium portal frames

The snow load was assessed based on an S_g value of 0.9kPa in accordance with AS/NZS1170.3 [6] as modified by the December 2008 revision to NZ Building Code section B1.

A static analysis was carried out to assess the frame forces generated by the gravity loads acting on the stadium roof. The results of the analysis show that the stadium portal frames have a snow loading capacity of 89% NBS for load case 1.2Dead load + Snow load.

6.5 Discussion of results

As determined from the qualitative assessment the seismic capacity of the original building was typically greater than 100% NBS but certain elements required detailed assessment. These checks have confirmed that the reinforced masonry walls of the original building have greater than 100% NBS capacity for out of plane seismic forces. This utilises a ductility factor $\mu = 2.0$ for flexural response and takes into account the additional restraint provided by the first floor installed into the Squash Courts in 1999 & 2008.

However, as part of this work, the original dividing wall was removed at first floor level leaving a 500mm long pier. This was the subject of correspondence between Christchurch City Council and Capital Programme Group in May 2008, from which it appears that some steel strapping was added. We presume that this was designed to achieve at least 100% NBS based on the then current Z value of 0.22, therefore utilising the current value of 0.3 would give a minimum 73% NBS result.

The stadium steel portal frames have a capacity of 95% NBS for seismic loading.

The stadium steel portal frames have been proven to have 89%NBS capacity for the snow load case. The limiting factor for the frame strength is lateral torsional buckling and the detailed assessment considered the ability of the existing purlins to provide restraint to the rafter top flange. If desired this could be improved by the introduction of longitudinal CHS members to provide additional restraint to the rafters.

The short precast concrete wall of the 1999 extension, oriented North-South on Grid 7, has greater than 100%NBS for in plane shear resistance, utilising a ductility factor of 1.25. The wall is connected to the foundation and adjacent ground floor slab, and the connection has an overall capacity of greater than 100% NBS, however if the adjacent slab is removed the dowel bars from the foundation would only achieve 50%NBS.

The East-West frames of the 1999 extension have 84% NBS capacity based on their flexural strength. However this is above the 67% minimum required and therefore no improvement work is required. A check on deflection due to ULS forces indicates a potential lateral eaves movement of around 200mm but we consider this to be an acceptable result as it is unlikely to cause damage to the covering sufficient to cause collapse or other safety risk. It may however cause some distortion with the potential to allow weather ingress but this is considered acceptable for such an extreme event.

7 Performance in the 22 February Christchurch Earthquake

The epicentre of the February 2011 magnitude 6.3 Christchurch Earthquake was located 5.5km South East of Pioneer Recreation and Sports Centre (see Figure 1). A strong motion recording of the ground motions from this earthquake was made by the Geonet network at a station (CMHS) that is located approximately 0.8km from the site. In view of their close proximity, these recorded ground motions are likely to be very similar to those experienced at the site.

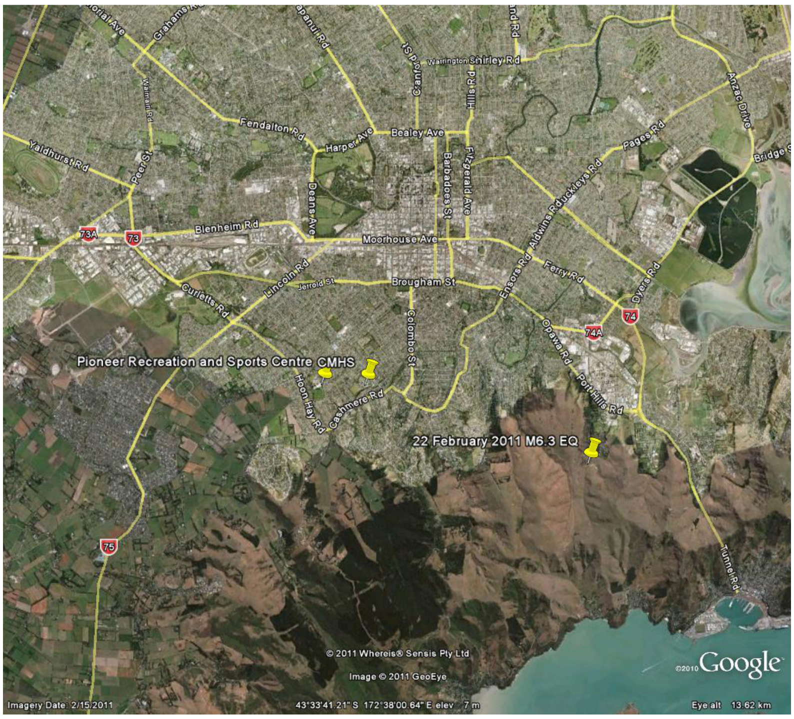

Figure 1: Relative locations of Pioneer Recreation and Sports Centre, Geonet station (CMHS) and the epicentre of the Christchurch Earthquake

Figure 2 shows the 5% damped response spectra for ground motions from the 22 February magnitude 6.3 Christchurch Earthquake recorded at the CMHS recording station, along with the current design 100% NBS standard spectrum.

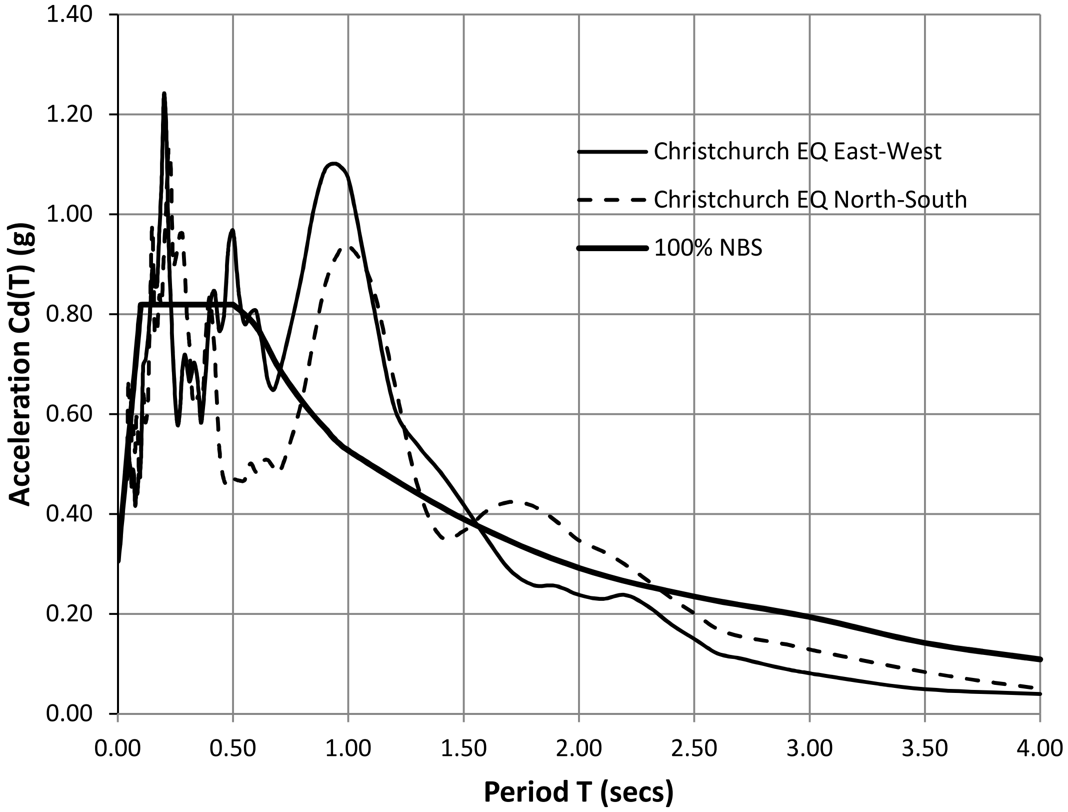

Figure 2: Acceleration response spectra from ground motions recorded by Geonet station (CMHS) near to Pioneer Recreation and Sports Centre, compared with the current design standard 100%NBS (IL 3) acceleration spectrum for the site

It can be seen that the North-South and East-West loads experienced by the site are generally of the same magnitude as 100% of the New Building Standard. The building sustained minor structural damage in the Christchurch earthquake comprising some cracking of masonry to the Stadium and minor horizontal separation of the Foyer ground level slab at the junction of the original building and 1999 Administration extension. The performance of Pioneer Recreation and Sports Centre under these quite severe loads is an indication that it is of generally robust, earthquake resistant construction

8 Conclusions

We conclude as follows:

a) The seismic capacity of the original stadium is 95% NBS, which is governed by the flexural capacity of the north-south portal frames.

b) The seismic capacity of the Pool Hall is 84% NBS, which is governed by the flexural capacity of the east-west portal frames.

c) A small pier at first floor level in the former squash courts was assessed to have a minimum seismic of 73% NBS.

d) The snow loading check on the stadium steel portal frames showed that they have a capacity of 89%NBS. It would be possible to improve this element to around 100%NBS if desired.

9 Recommendations

a) Minor repair works should be carried out on cracking to reinforced concrete masonry walls in the Stadium and opening of the slab joint in the foyer.

10 References

[1] Pioneer Recreation and Sports Centre*, Detailed Engineering Evaluation, Stage One Qualitative Report*, prepared by Opus International Consultants for Christchurch City Council, September 2011.

[2] *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, draft prepared by the Engineering Advisory Group, Revision 5, July 2011.

[3] *NZS 1170.5:2004, Structural Design Actions, Part 5 Earthquake Actions New Zealand*, Standards New Zealand.

[4] *NZS 3404:1997, Steel Structures Standard*, Standards New Zealand.

[5] *NZS 3101:2006, Concrete Structures Standard*, Standards New Zealand.

[6] *AS/NZS 1170.3, Structural Design Actions, Part 3 Snow Actions*, Standards New Zealand.

Appendix A - Photographs

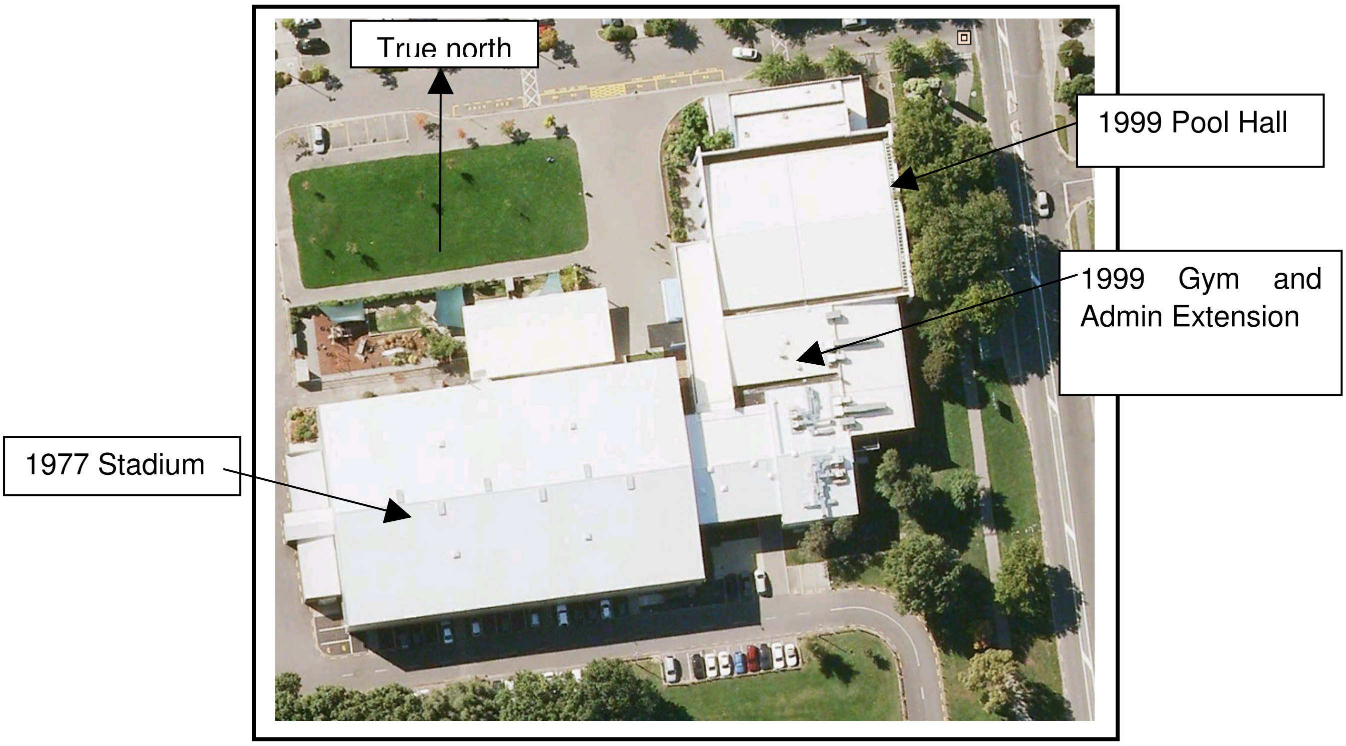

Photo 1 – Site location

Photo 2 – Pool Hall

Photo 3 – Stadium

Photo 4 – Opening of joint in north west corner of Stadium